EARTHQUAKE ENGINEERING AND STRUCTURAL DYNAMICS, VOL. 25, 1109–1137 (1996)

# ANALYTICAL AND FIELD EVIDENCE OF THE DAMAGING EFFECT OF VERTICAL EARTHQUAKE GROUND MOTION

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## SUMMARY

The paper is an attempt to collate field evidence and results from dynamic analysis on possible structural effects of strong vertical ground motion. Observational evidence from three earthquakes are presented and assessed with regard to failure modes of buildings and bridges attributable to high vertical earthquake forces. Analytical results from previous studies for the same structural types are reviewed. These collectively confirm that structural failure may ensue due to direct tension or compression as well as due to the effect of vertical motion on shear and flexural response.

KEY WORDS: seismic response of buildings and bridges; vertical earthquake motion

## 1. INTRODUCTION

There is now a body of evidence indicating that the results of earlier strong-motion data analysis underestimate the vertical-to-horizontal peak acceleration ratio ($V/H$), at least for sites in the near-field of large earthquakes, while the opposite is true in the far-field.[1] The main reason for the apparent underestimation, as well as inconsistencies observed between the results reported by previous researchers, is attributed to the fact that regression in the context of attenuation relations was performed for the entire range of epicentral distances and magnitudes, rather than concentrating on distinct intervals in the epicentral distance range. Hence, unavoidable bias of the results towards the best represented intermediate-field ranges, where the distance-sensitive vertical component has attenuated significantly, is induced.[2] Recent analysis of strong-motion data conducted in line with the above observation indicates that in the vicinity of moderate to strong earthquakes the $V/H$ ratio often exceeds unity and hence violates on average by a factor of at least 30 per cent modern codified values[2] (same spectral shape is assumed). In addition, recent research has also shown that the attenuation of the $V/H$ spectral ratio can exceed by far the 2/3 spectral scaling rule and values as high as 1.7 for short source distances and short periods are not uncommon.[3] Although the latter study is conducted for the Northridge (1994) and the Loma Prieta (1989) events, as well as for SMART-1 array data, the similarity of the observed trends suggest that this behaviour is universal.[3]

In opposition to the above, it is commonly argued that vertical strong-motion acceleration peaks are insignificant for damage potential due to their low energy content. It is indeed the case that the energy content of the vertical component is significantly less than that of the corresponding horizontal component.[4,5] However, such a basis for dismissing the vertical component is inadequate, since the horizontal energy content is dominated by long period pulses which are non-existent in vertical strong-motion records. What is of significance in earthquake response is the relationship between structural and excitation periods. Hence, the strong-motion energy stored in this frequency range is the absolute criterion to decide on the importance of the components of earthquake motion.[4,5]

Furthermore, it is argued that a large safety factor against gravity loads exists in properly engineered structures and hence the probability against failure from vertical earthquake forces is low. Notwithstanding,

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CCC 0098-8847/96/101109-29

*Received 26 March 1996*
*Revised 8 May 1996*

some of the field examples presented hereafter point towards damage caused directly by an increase in axial compressive forces. Moreover, in some cases the neglected fluctuation in axial forces may lead to flexural and shear failures that would not have occurred in the absence of severe vertical earthquake motion. The evidence and commentary given in this paper seek to resolve the above conflict by re-examining observational and analytical information.

## 2. FIELD EVIDENCE

### 2.1. *Effect on buildings*

*2.1.1. The Kalamata, Greece, earthquake of 13 September, 1986.* Of particular interest is the case of the moderate Kalamata earthquake. This $M_s = 5{\cdot}7$ event had an epicentre located less than 9 km from the town centre and a focal depth of 7 km.[6] This type of shallow near-field event, having a relatively short recurrence interval, is a major threat to densely populated regions, especially in Europe and other intraplate zones. The cases of the Skopje (Former Yugoslavia, 1963), Managua (Nicaragua, 1972), Thessaloniki (Greece, 1978), E1-Asnam (Algeria, 1980), San Salvador (E1-Salvador, 1986), and Spitak (Armenia, 1989) earthquakes have repeatedly identified their potential destructive power. What is more important within the context of this study is their association with strong vertical ground motion, attributable to the proximity of the source of energy release, and the need to identify the role of such motion in the observed damage which is commonly ignored in design. The Kalamata earthquake gave rise to strong-motion recordings with a $V/H$ ratio of 0·81 and 1·26.[7] Strong vertical motion, as high as 0·38 g for the ITSAK station, was verified during field observations, where items displaced horizontally without any evidence of friction at the interface. An example is shown in Figure 1. In another case, a RC pedestal cracked horizontally at mid-height, thus signifying a possible tensile force.

In terms of structural damage inflicted on RC buildings, Elnashai *et al.*[6-8] report an unusually high number of symmetric compression and shear-compression failures in columns and shear walls, as opposed to bending failures. Such failures are even observed in buildings with a soft ground storey where bending failure is naturally expected. Although many of these can be attributed to poor detailing and construction errors, it is suspected that the strong vertical component also played a significant role. It is considered that the

Figure 1. Evidence of strong vertical motion. A bench which has displaced horizontally without friction at the interface

variation of vertical forces inevitably gave rise to a reduction in shear strength due to loss or reduction of the concrete contribution. In the opinion of the writers this led to the observed shear failures in walls and columns. Moreover, the increase in axial compressive forces due to vertical motion in combination with poor detailing also led to a large number of compression and compression-shear failures in walls and columns, in cases where the transverse shear capacity was not exceeded by demand. These were often manifested by symmetric buckling of the reinforcement or by X-type shear failure at mid-height of columns or walls. An example is shown in Figure 2 where compressive failure of a first storey RC column took place despite the presence of an incomplete infill panel which would preferentially induce a shear failure in the short column.

*2.1.2. The Northridge, California, earthquake of 17 January, 1994.* The $M_s = 6{\cdot}7$ Northridge, California, 17 January 1994 earthquake is of milestone significance in this study, since it is the first earthquake where many instruments recorded unusually high vertical accelerations and structural damage of modern buildings attributable to vertical motion was observed. The vertical accelerations were as high as 1·18 g, while $V/H$ ratios were as high as 1·79. The event has been classified as one of the most destructive earthquakes affecting the west coast of the U.S.A.[9] Table I shows a selection of recorded strong-motion, both in terms of vertical peak acceleration and $V/H$ ratio.

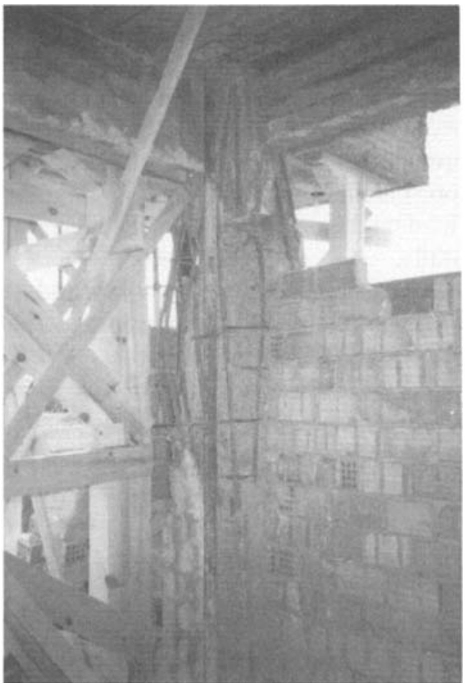


Figure 2. Compressive column failure in a residential building in Kalamata. The presence of the incomplete infill panel did not induce shear failure in short column

Table I. Northridge records possessing a strong vertical component. Data from Broderick *et al.*[10]

| Station | Epicentral distance (km) | Vertical PGA (g) | Horizontal PGA (max) (g) | $V/H$ |
|---|---|---|---|---|
| Tarzana, Cedar Hill Nursery | 5 | 1·18 | 1·82 | 0·65 |
| Arleta, Nordhoff Avenue Fire Station | 10 | 0·59 | 0·35 | 1·69 |
| Sylmar, County Hospital | 16 | 0·60 | 0·91 | 0·66 |
| Newhall, LA County Fire Station | 20 | 0·62 | 0·63 | 0·98 |

In general, reinforced concrete buildings suffered significant structural damage during the Northridge earthquake. In particular, many instances of intermediate storey damage or even collapse were observed.[9–11] These failures can be attributed to a number of reasons, namely higher mode response, abrupt changes of stiffness and strength in elevation, or lack of a capacity design approach in higher storeys. However, there is concern that many of them were induced in a brittle fashion by direct compressive failure or reduction of shear strength and ductility supply due to the variation of axial forces arising from the action of vertical motion.

Of instrumented buildings closest to the epicentre the Holiday Inn Hotel in Van Nuys was located approximately 7 km east of the epicentre. This 7-storey RC frame experienced a peak horizontal acceleration of 0·47 g and a peak vertical acceleration of 0·30 g at the base which resulted in serious structural damage in several columns of the third floor,[9] as shown in Figure 3. The maximum recorded horizontal acceleration at the roof was 0·59 g, i.e. an amplification of only 1·25. Inspection of the acceleration time-histories recorded in the building verified that they are all in phase, hence indicating first mode response in the horizontal direction. Moreover, the same records show that no torsional response took place.[9] Therefore, there is very high probability that the shear-bond splitting failure which was only observed in the third storey was induced by a reduction of shear capacity due to vertical motion, as explained previously. It is here appropriate to note that for first mode vertical response the reduction of axial force in columns or walls is more profound for higher storeys, since it represents a larger relative change of the pre-existing static axial load. Hence, a larger instantaneous reduction of column shear capacity is expected for upper floors. This, combined with higher shear force demand in lower storeys justifies the intermediate storey failure observed.

An example of intermediate storey failure which resulted in collapse was the case of the Kaiser Permanente administrative building situated on the Balboa Boulevard in Northridge. The 5-storey mixed wall-frame RC building lost the entire second storey due to shear failure of the columns and beam–column joints. This is shown in Figure 4. In addition, poor detailing in the beam–column joints lead to complete collapse.[9] The detrimental effect of reduced axial load on the ductility and shear capacity of RC joints is a known fact which has been demonstrated experimentally.[12]

Vertical earthquake motion does not only influence ductility and shear capacity supply relevant to transverse response. Compressive failure can also occur in vertical bearing elements such as walls and columns. In view of current design procedures, interior columns are more vulnerable in this respect, since

Figure 3. Shear-bond splitting failure in 3rd storey of Holiday Inn Hotel in Van Nuys

Figure 4. Second storey collapse of Kaiser Permanente building in Balboa Boulevard at Northridge. Photograph courtesy of Earthquake Engineering Research Institute

their design is not greatly influenced by increased axial loads due to overturning. However, assuming a uniform axial stiffness distribution in plan, vertical excitation causes a uniform increase in the axial force response of all columns in the same storey irrespective of their position. An example of interior column failure is the 3-storey parking structure of the California State University at Northridge, a modern structure only 18 months old at the time of the earthquake.[13,14] The structural system comprised a perimeter ductile precast moment resisting frame which was responsible for accommodating the transverse earthquake loads. The internal frame, comprising precast beams and columns, constituted the gravity load resisting system. During the earthquake some of the interior columns suffered total collapse due to compression failure, while the lateral load resisting system did not exhibit signs of distress arising from horizontal earthquake-induced forces.[10,14] Damage to this system was secondary and was due to collapse of interior columns and subsequent inward bending of the lateral force resisting system, which exhibited extremely high rotational ductility,[10,14] as illustrated in Figure 5. Such a failure mode suggests a very substantial increase of the axial forces acting on the gravity system. This was most likely induced by direct action of the vertical component or from possible vertical vibrations of the floor system. This mechanism was also manifested by shear-compression failure in interior columns of a cast-in-place garage in Sherman Oaks.[11] A damaged column of this structure is shown in Figure 6.

An interesting case of punching shear failure in waffle slabs occurred in the upper two floors of the Bullocks store in Fashion Island Mall. This collapse almost certainly occurred as a result of the high vertical accelerations experienced in the epicentral region which triggered strong vertical oscillations of the flat slab.[14] In the particular structure, the majority of the columns supporting these slabs showed only minor damage which suggests a limited effect of horizontal motion. The failure mode is shown in Figure 7. Punching shear failure was also observed in the Four Seasons Hotel in Sherman Oaks and in the flat slabs supporting a number of timber apartments.[10,11] Moreover, the severe effect of the vertical component on floor oscillations can also be observed in the Northridge Fashion Centre north-west parking structure. The simply supported double-tee floors of this building experienced severe vertical oscillations which resulted in shear cracking as well as total collapse of the floor system.[14] This is illustrated in Figure 8. The mode of damage is a consequence of vertical excitation since the floor system was independent of the lateral load resisting system.

Figure 5. Collapsed CSUN parking structure. The perimeter frame does not show signs of distress where gravity system has not failed. Photograph courtesy of Earthquake Engineering Research Institute

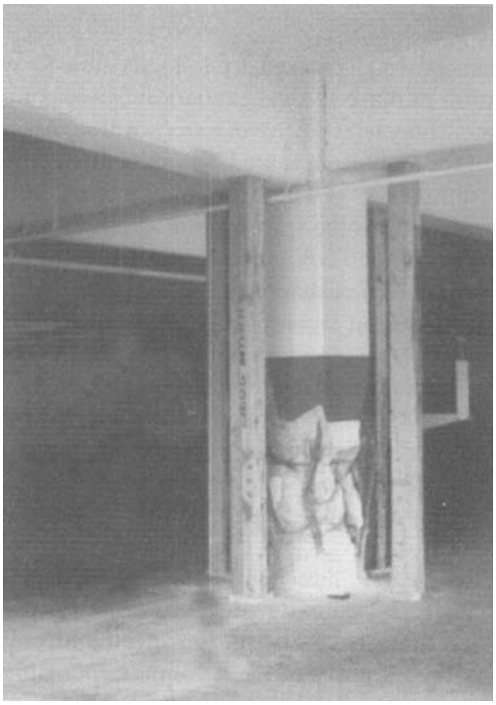


Figure 6. Shear-compression failure in column of cast-in-place garage in Sherman Oaks. Photograph courtesy of National Center for Earthquake Engineering Research

The most unexpected damage form reported from Northridge is the crack patterns observed in over 100 steel rigid moment resisting frames at the beam–column connection and column web.[10] Although no steel buildings collapsed, the cost associated with repair and lost revenue is enormous. The observed failure of welded beam–column connections was mostly due to fracture of the weld, but sometimes also involved the

Figure 7. Punching shear failure in waffle slabs of the Bullocks store. Note intact columns and original floor levels. Photograph courtesy of Earthquake Engineering Research Institute

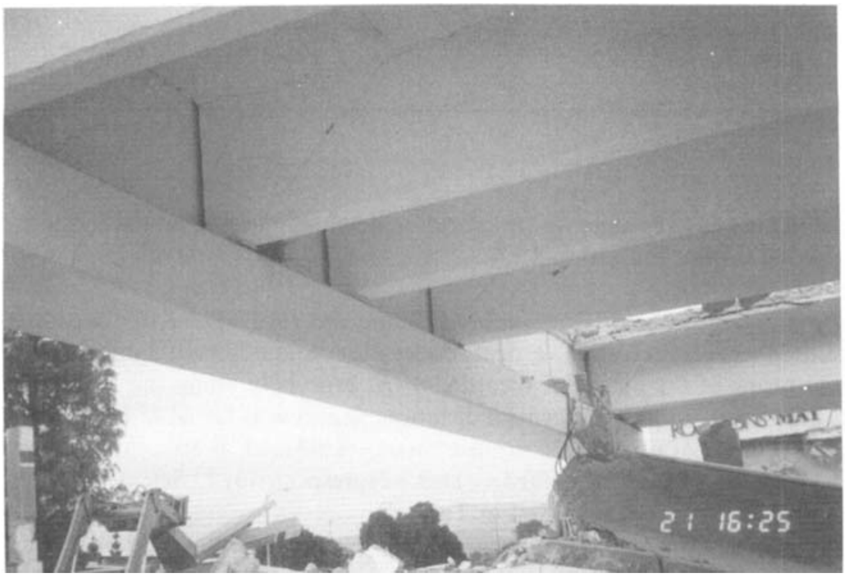



Figure 8. Northridge Fashion Centre north-west parking structure. Note cracks in floor and transverse beams. Photograph used with kind permission of Dr Said Hilmy

fracture of the column flanges. In some instances fracture was observed away from the connection within the column web. The former failure model is shown in Figure 9. It is thought that these cracks are a result of low-cycle fatigue in combination with poor detailing. However, in order to justify the number of post-yield cycles on the connection the contribution of vertical beam vibrations, due to vertical excitation, or less likely, that of cumulative earthquake damage need to be considered.[10] It is herein postulated that beam vibrations

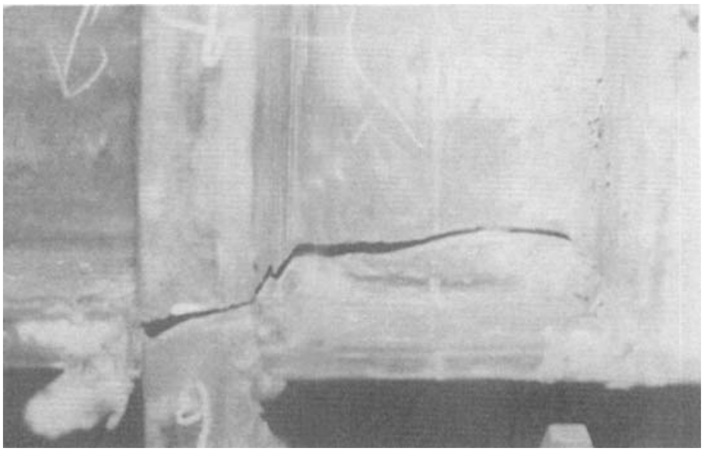


Figure 9. Typical beam-column connection fracture observed following the Northridge earthquake. Photograph courtesy of EQE International

Table II. Kobe records possessing a strong vertical component. Calculated using data from Reference 16

| Station | Epicentral distance (km) | Vertical PGA (g) | Horizontal PGA (max) (g) | $V/H$ |
|---|---|---|---|---|
| Kobe, Port Island Array | 20 | 0·57 | 0·35 | 1·63 |
| Kobe, University | 25 | 0·43 | 0·31 | 1·39 |
| Kobe, JMA station | 18 | 0·34 | 0·84 | 0·41 |

under vertical motion may have exaggerated the rotation demand imposed on the connections by horizontal vibrations, as discussed further in Section 3.1.4.

*2.1.3. The Hyogo Ken Nanbu, Japan, earthquake of 17 January, 1995.* The Kobe earthquake was similar to the Northridge event in terms of recorded vertical accelerations and observed damage. However, this event is probably unique in terms of the extent of recorded $V/H$, especially along the fault rupture zone. Strong vertical motion was recorded at large epicentral distances. Stations as far as 45 km from the epicentre (thus emphasising the importance of using fault rupture distances instead of epicentral distances) gave vertical accelerations and $V/H$ ratio of the order of 0·33 g and 1·21, respectively.[15] Selected peak accelerations from records with strong vertical motion are given in Table II.

Preliminary field investigation also points towards failure patterns influenced by vertical motion, such as column shear failure not at the location of maximum shear and an increased number of column compressive failures. Most of the intermediate storey collapses are concentrated in the Chuo Ward of Kobe and account for over 10 per cent of the collapsed or severely damaged buildings in the area.[15] It is noted here that this is the area where the largest vertical accelerations were recorded and the records shown in Table II obtained. The majority of buildings with intermediate storey collapse were 6 to 12 storeys in height. Maximum spectral amplifications of nearby records at sites with similar ground conditions are above 1·0 s and hence second mode response would have not been significant. Typical examples of intermediate storey collapse are shown in Figures 10 and 11. These particular examples are selected from a wide variety of available cases and every effort is made here to eliminate cases where change in stiffness, construction material in elevation, pounding with adjacent buildings, presence of elevated corridors or plan irregularity have contributed to the damage.

Figure 10. A 10-storey building with a collapsed 3rd floor in Kobe city. No tilt or misalignment of the collapsed block relative to the base is observed

Figure 11. A 12-storey building with a collapsed 5th floor. Note squashing and out of plane buckling of shear wall

Some buildings suffered shear-compression failure of several columns along their height. These failures are of comparable extent along the height which suggest a predominant axial response. An example is a 4-storey building located along Route 2 in the centre of eastern Nada Ward[15] which is shown in Figure 12. An internal first storey column of the building which failed in compression is also shown in Figure 13.
In terms of vertical response several instrumented buildings in the area showed significant amplification in the vertical direction. Some records from instrumented buildings are shown in Table III.

It is observed that for a wide range of building and material types the vertical response amplification is higher than the corresponding horizontal and is not influenced significantly by building height. The above can be attributed to two factors. First, damping in the vertical direction is less than in the horizontal due to the absence of an efficient energy dissipating mechanism. The second reason is quasi-resonant response for


Figure 12. A 4-storey building in Kobe with shear-compression failure in all internal columns. Photograph used with kind permission of Dr Minehiro Nishiyama

Figure 13. Compressive failure of internal column of the building shown in Figure 12. Photograph used with kind permission of Dr Minehiro Nishiyama

Table III. Peak response of instrumented buildings and response amplification in the vertical and horizontal directions. Data obtained from Reference 15

| Building | Epicentral distance (km) | Recorded accelerations (g) | | | | Amplification | |
|---|---|---|---|---|---|---|---|
| | | *V*: GL | *H*: GL | *V*: Roof | *H*: Roof | *V* | *H* |
| Takami residence 31-storey reinforced concrete | 43 | 0·26 | 0·27 | 0·44 | 0·31 | 1·69 | 1·15 |
| Osaka Institute of Technology Building No. 6, 16-storey steel | 52 | 0·13 | 0·19 | 0·20 | 0·27 | 1·54 | 1·42 |
| Osaka Institute of Technology No. 7, 11-storey composite | 52 | 0·13 | 0·19 | 0·26 | 0·30 | 2·00 | 1·58 |

Figure 14. Column brittle tensile failure observed in a steel mega-truss at Ashiya-hama

a wide range of building frames in the vertical direction, given that buildings are generally very stiff in the vertical direction and vertical strong-motion is predominantly composed of high frequency pulses. Moreover, building frames are designed to a fixed safety factor against vertical loads and the relationship between strength and stiffness in the vertical direction is linear, which suggests that the vertical period is insensitive to the building height. Measurement of structural response (Table III) also dismisses the argument that vertical motion is "impulsive" in nature and does not cause significant structural response.

Severe cracking in steel structures was also observed not only at beam–column connections, but also in box column members.[15] In the latter case, fracture involved a horizontal plane with a separation of up to 20 mm. Also, the absence of bending deformation of the plates comprising the box column sections suggest a predominant axial response involving tension.[15] This mode of failure is illustrated in Figure 14. Some of these fractures have also lead to steel building collapses, probably observed for the first time. However, it must be noted here that the use of brittle steel in combination with poor detailing was responsible for some of the observed failures.[15,17]

Of particular interest herein is the failure of members of the mega-truss comprising the structural system of the Ashiya-hama complex. Since the truss system ensures that no bending action exists, observed horizontal

severance of vertical members can be attributed to vertical motion, or to a combination of more components, the action of which induced tension that exceeded by far the gravity loads.[18]

*2.2. Damage to highway bridges*

*2.2.1. Northridge: Bull Creek Canyon Channel bridge.* Several Highway bridges suffered serious damage, as well as total or partial collapse during the Northridge earthquake. One of the bridges nearer to the epicentre which suffered substantial damage was the Bull Creek Canyon Channel Bridge which was completed in 1976. The bridge carries the SR118 over a channel approximately 9 km north east of the epicentre. The three spans of the RC bridge are approximately 30 m long and are supported on two multi-column bents. The spans are bridged by a continuous prestressed 13-cell box girder. The deck also features a longitudinal expansion joint which separates the bridge in two parts, namely North and South. The skew of the supports ranges from 36° at the West abutment to 47° at the East abutment. The piers supporting the bridge are 1·2 m wide and are of octagonal cross-section. The confinement to these piers is provided by 16 mm spiral bars arranged at 75 mm centres at the pier ends and at 300 mm centres in non-critical sections.[9] A view of the bridge after the earthquake is shown in Figure 15.

All of the bridge piers at bent 3 (total 10 piers) suffered severe damage at their base. In particular, fractured transverse reinforcement, buckling of longitudinal column bars, core disintegration and significant axial shortening was observed, as depicted in Figure 16. On the contrary, only the two southernmost columns of Bent 2 failed in a similar manner below the transition of hoop reinforcement near the column top. It has been argued that the failure of bent 3 was induced by the formation of plastic hinges at the column base. Also, the presence of a structurally connected wall aligned with the channel and the bent was blamed for reducing the clear span of the piers and hence forcing hinging in the less confined zone.[9]

In the absence of substantial transverse displacements and of deck distress, the above explanation does not provide a full justification of the observed damage. In particular, the above interpretation does not answer the questions: (i) why the piers of bent 3 all formed plastic hinges only at their base and not at the top as well, since the bending moments at the two ends are comparable and the reinforcement details identical?, (ii) why was the failure of all piers along bent 3 of similar extent, whilst transverse loading would have imposed higher axial forces on external piers? and (iii) why the columns of bent 3 did not fail in shear in view of the reduced clear span? In view of the exceptionally high vertical accelerations recorded in neighbouring stations (Rinaldi: $a_v = 0{\cdot}85$ g, $a_h = 0{\cdot}84$ g, 1 km to the north; Arleta: $a_v = 0{\cdot}59$ g, $a_h = 0{\cdot}35$ g, 3 km to the south east)

Figure 15. Bull Creek Canyon Channel Bridge. Significant vertical deformations can be observed at mid-span. Photograph courtesy of National Center for Earthquake Engineering Research

Figure 16. Compression failure and bulging of piers along bent 3. Bull Creek Canyon Channel Bridge. Photograph courtesy of Earthquake Engineering Research Institute

and the severe vertical deformations and bulging experienced by the piers, the magnification of axial loads by vertical earthquake motion provides a better explanation for the mode of failure. In particular, severe buckling of the reinforcement and core disintegration as a result of large axial forces is preferentially induced at the regions where spacing of the hoops is large. Moreover, the difference in the intensity of damage between the bents can be attributed to the different span lengths supported and hence different amplification of vertical response in the associated piers.

*2.2.2. Northridge: La Cienega-Venice Undercrossing Collector Distributor 36.* The Collector Distributor 36 forms part of a pair of off-ramps from the eastbound carriageway on the I-10 freeway at La Cienega-Venice Undercrossing and was designed and constructed between 1962 and 1965. The ramp was located some 25 km to the southeast of the epicentre. From the bifurcation point just to the west of bent 5 of the Undercrossing, the RC ramp was carried first over the multi-column bent 5, then over three single piers (6, 7 and 8) and finally over the pier wall of bent 9 to the east abutment. The deck consisted of a 3-cell continuous box girder which was rigidly connected to the supporting structure.[10]

No visible damage was inflicted on either the ramp deck or the abutment. The piers supporting the bridge experienced varying levels of damage, the extent of which was inversely proportional to pier height. In particular, pier 6 experienced spectacular failure and was the most damaged of all the columns supporting the ramp.[10] Shear failure in the transverse direction (north–south) occurred in the lower half of the pier. The concrete cover completely spalled over the height and the concrete core disintegrated. Moreover, all the reinforcement bars buckled symmetrically and the transverse hoops opened, leaving the pier with permanent axial deformation, as shown in Figure 17. There is evidence that the collapse of this pier is partly attributed to the instantaneous reduction of shear strength caused by vertical motion and the resulting fluctuation of the pier axial load, as substantiated in Section 3.2.1 of this study. The nearest strong-motion records suggest a maximum horizontal peak ground acceleration of up to 0·3 g (east–west, north–south components are far less) and peak vertical acceleration of up to 0·22 g.[9]

*2.2.3. Kobe: Failures of highway bridge piers.* From the numerous bridge failures in Kobe, a particular failure mode has been observed which is strongly attributable to an increase in pier axial compressive forces caused by vertical ground motion. In particular, compression failures have been induced, manifested by

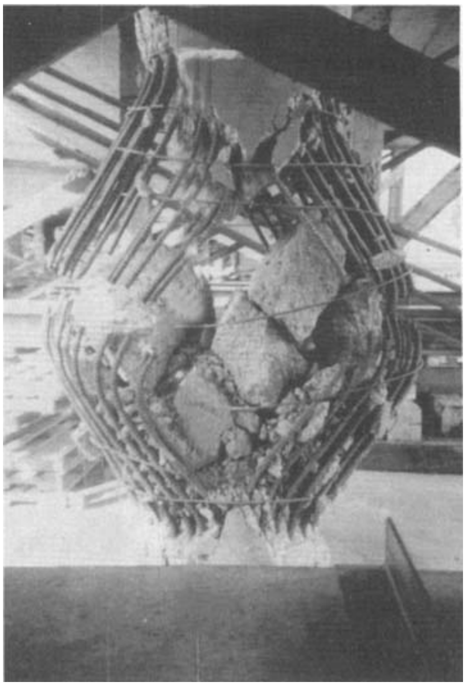


Figure 17. Pier 6 of Collector Distributor 36. Damage induced by shear failure caused by reduction of axial load

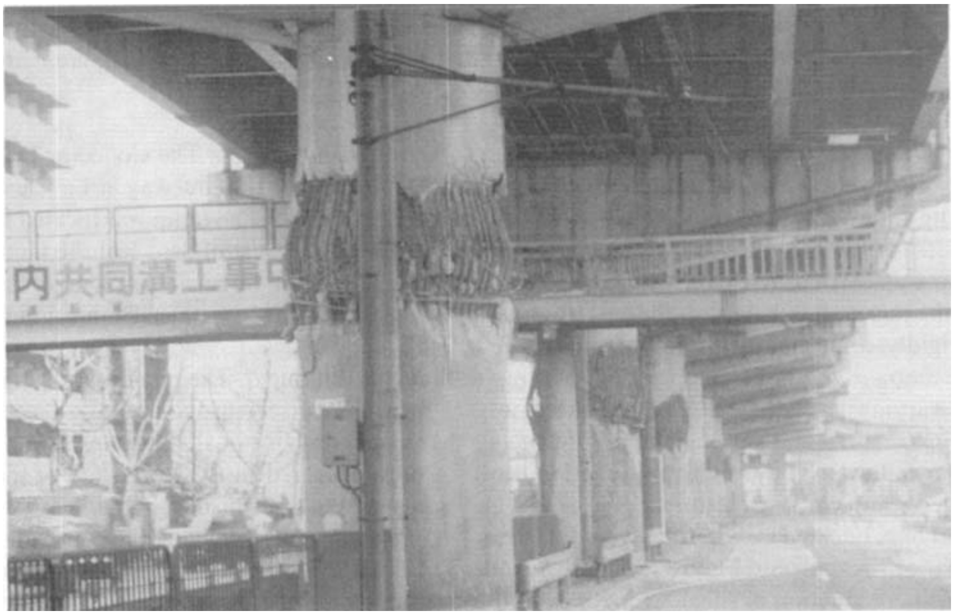



Figure 18. Series of compressive failures along piers of the Hanshin Expressway. Photograph courtesy of EQE International

symmetric outward buckling of longitudinal reinforcement and crushing of the concrete at mid-height of circular piers. Limited, or no, bending rotations have been observed in the crushed zones. Such failures are observed along a series of piers at sections of the Hanshin Expressway and the Port Island Crossing, as demonstrated in Figures 18 and 19. For the former bridge, compression failure was initiated close to where

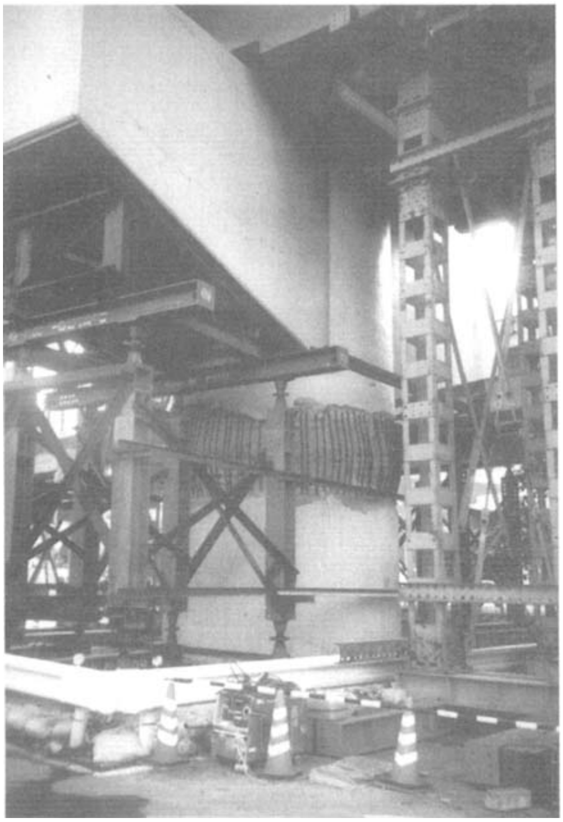


Figure 19. Compressive failure of a pier of Port Island Crossing. Note distress on pier head

layers of longitudinal pier reinforcement were terminated, while for the latter failure occurred below the pier head where stress concentration caused by a reduction of cross-section is expected.[16] Moreover, in the case of the Port Island Crossing, signs of distress were also visible in the pier head, as shown in Figure 19, thus emphasising the argument that the damage was induced due to vertical strong-motion.

Finally, compression-dominated failure has also been reported for two more bridges, namely the Meishin Expressway and the Second Shinmey Overbridge.[19] The former was of continuous cast-in-place construction and consisted of wall-type piers. Several piers suffered compressive failure manifested by bulging and significant axial shortening, as shown in Figure 20. Buckling of longitudinal reinforcement was also observed. The latter bridge comprised continuous steel box-girder superstructure supported on pairs of squat piers connected by transverse beams. The piers of the bridge exhibited an unusual mode of failure manifested by bursting of the pier along the length and fracture of transverse reinforcement. The mode of failure is shown in Figure 21.

### 2.3. *Remarks*

The field observations presented above, for buildings and bridges, collectively indicate that there are failure modes that cannot be explained by considering shear and flexural capacities only. In such cases, it is likely that axial overstressing provides a more convincing justification of the observed damage.

Evidence is also emerging that the very high death toll in the Sakhalin-Neftegorsk earthquake of 28 May 1995 was greatly influenced by vertical earthquake motion. In a report by Eisenberg,[20] it is stated clearly that crushing of bearing walls due to very high vertical ground acceleration caused the collapse of many

Figure 20. Compressive failure of wall-type pier of Meishin Expressway. Photograph used with kind permission of Professor Atsuhiko Machida

Figure 21. Compressive failure of squat piers of Second Shinmey Overbridge. Photograph used with kind permission of Professor Chitoshi Miki

residential RC buildings. Although no near-field recordings were obtained, evidence provided by survivors, and in particular uplifting of objects at 2nd and 3rd floors of RC buildings, confirm strong vertical motion.

On a one-by-one basis, mechanisms other than axial distress may be forwarded. However, it is more logical to re-assess all the above cases collectively, since it may be that the common feature is indeed an increase, or a decrease, in axial load that has not been accounted for in the design process. This is particularly emphasised by the analytical results presented in subsequent sections which confirm the existence of high vertical response.

## 3. ANALYTICAL EVIDENCE

### 3.1. Analysis of buildings

*3.1.1. Vertical periods.* Of fundamental importance are the natural periods for vertical vibration of buildings. Buildings seem to be much stiffer in the axial than in the transverse direction and hence possess shorter periods in the vertical direction. Papadopoulou[21] indicates that for RC moment resisting frames the ratio of vertical-to-horizontal fundamental periods varies from 7 to 2·5 for a range of storeys from 8 to 1. The results of this study are summarized in Table IV.

Eigenvalue analysis of a 3-bay 8-storey coupled wall-frame RC structure designed according to EC-8,[22] and recently studied at Imperial College,[23] identified vertical periods of 0·075 s or less. This compares with a horizontal period of 0·534 s.[22] The above periods for RC buildings are calculated on the basis of compressive column stiffness, while no account is taken of the reduction of this stiffness due to cracking when the column is in tension.

Similar patterns are also observed for steel buildings. Papaleontiou and Roesset[24] have studied four 3-bay steel moment resisting frames having spans of 4·5–8·4 m. It is noted that these structures are taken from several other studies and are not consistent in terms of their design. In particular, the 4-storey and the 10-storey frames are comparatively much more flexible in the horizontal direction than the other two and clearly this affects the ratio of vertical to horizontal periods. However, these examples can still be used to indicate a very broad trend of this important ratio, as applied to steel moment resisting frames. The relationship between vertical and horizontal periods is shown in Table V.

The above values support the comments made earlier, consistent with field observations, that vertical periods are not significantly influenced by building height and lateral stiffness. This suggests that a wide range of buildings experience approximately the same dynamic amplification during vertical excitation.

Table IV. Relationship between first mode horizontal and vertical periods for RC buildings. Reproduced using data from Papadopoulou[21]

| Number of floors | Horizontal period (s) | Vertical period (s) | Ratio |
|---|---|---|---|
| 1 | 0·1 | 0·040 | 2·50 |
| 2 | 0·2 | 0·064 | 3·13 |
| 3 | 0·3 | 0·082 | 3·66 |
| 4 | 0·4 | 0·091 | 4·40 |
| 5 | 0·5 | 0·099 | 5·05 |
| 6 | 0·6 | 0·106 | 5·66 |
| 7 | 0·7 | 0·114 | 6·14 |
| 8 | 0·8 | 0·120 | 6·67 |

Table V. Relationship between first mode horizontal and vertical periods for steel buildings. Reproduced using data from Papaleontiou and Roesset[24]

| Number of floors | Horizontal period (s) | Vertical period (s) | Ratio |
|---|---|---|---|
| 4 | 1·0 | 0·16 | 6·25 |
| 10 | 2·22 | 0·20 | 11·10 |
| 16 | 1·54 | 0·19 | 8·11 |
| 20 | 2·27 | 0·25 | 9·08 |

What causes additional concern is the combination with the results of studies of predominant periods found in vertical strong-motion records. The constant amplification range for vertical strong-motion records is found to lie between periods of 0·05 s and 0·15 s.[4,5] This combined with the confirmed severity of near-field vertical strong-motion in terms of peak ground acceleration suggests that large dynamic axial forces, acting both upwards and downwards, should be expected in the near-field. Analytical studies supporting this statement are discussed below.

*3.1.2. Effect on axial forces and displacements.* Analysis of lumped parameter MDOF structural models employing bilinear stiffness characteristics in tension and compression applicable to RC column behaviour[21] indicates that strong vertical motion can lead to column tension. In the latter reference, a study involving a range of earthquake records and multi-storey buildings was undertaken. Initial gravity loads were explicitly

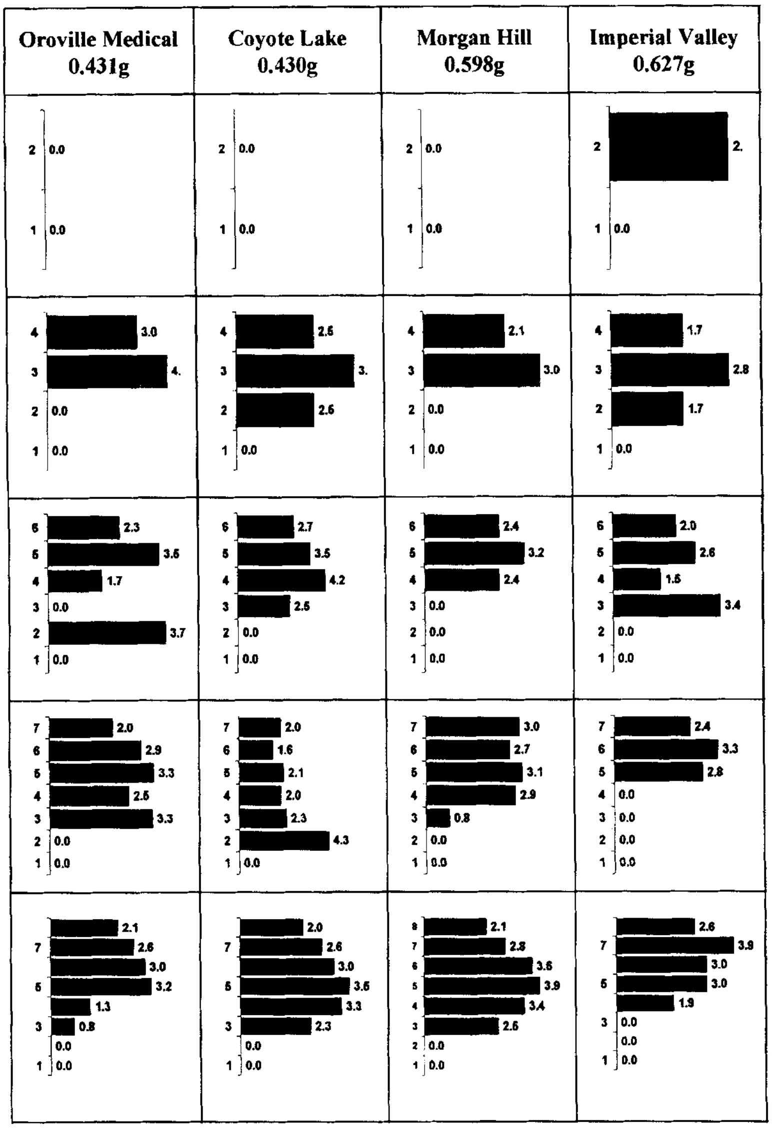



Figure 22. Pattern of maximum tensile displacements along the height of multi-storey RC buildings subjected to various vertical earthquake motions. Figures on the left of the graph indicate the storey number, while figures on the right indicate the maximum tensile displacement in mm corresponding to that storey. Reproduced using data from Papadopoulou[21]

considered in the nonlinear analysis. The study, which involved buildings having a uniform distribution of stiffness and mass with height, indicated that intermediate and the top storeys are more likely to undergo tensile deformations, depending on the relationship between the building and strong-motion periods, as well as the intensity of ground shaking. The results presented in Figure 22 show that for the buildings and records examined column tension in upper storeys always occurs for peak ground acceleration exceeding 0·43 g and for buildings having more than two storeys, even though horizontal motion is not considered in the analysis.

Papaleontiou and Roesset[24] performed linear time-history analysis of the steel buildings mentioned in Section 3.1.1 using the 1989 Loma Prieta record from Capitola ($a_v = 0{\cdot}52$ g; $a_h = 0{\cdot}47$ g). The analysis did not involve gravity loads. The maximum compressive ( $-$ ) and tensile ( $+$ ) axial forces for exterior columns, where the effect of overturning moments is larger, are shown in Table VI.

It is observed that the axial forces caused by vertical motion, having a comparable magnitude to horizontal motion, are larger than the corresponding forces due to transverse motion in most cases. This pattern is always more significant for upper floors than for lower floors. Still, for a 10-storey steel frame almost 50 per cent of the axial force variation in the ground floor exterior columns comes from contributions of vertical motion. This contribution is ignored in routine design. In interior ground floor columns the axial force variation arising from vertical motion is even more significant, since the effect of overturning moments is minimal.

Moreover, nonlinear dynamic analysis of an 8-storey, 3-bay moment resisting RC frame designed according to UBC, has lead to the confirmation of the occurrence of net tensile forces and displacements, thus dispelling the question of high frequency excitation often used to support the insignificance of the vertical component.[25] The results for the Imperial Valley Centro-6 motion are shown in Table VII.

A substantial increase of compressive axial forces is also reported. This is indicated in Table VIII for the response to the same motion.

Table VI. Effect of vertical motion on axial force response of steel frames. Calculated using data from Papaleontiou and Roesset[24]

| | Axial forces (kN) | | | | Contribution of vertical motion to total axialforce (%) | |
|---|---|---|---|---|---|---|
| No. of storeys | Roof $H$ | Roof $H+V$ | Ground $H$ | Ground $H+V$ | Roof | Ground |
| 4 | $-22/+40$ | $\pm 110$ | $\pm 200$ | $\pm 450$ | 72 | 56 |
| 10 | $\pm 22$ | $\pm 150$ | $\pm 490$ | $\pm 850$ | 85 | 42 |
| 16 | $-58/+80$ | $\pm 290$ | $\pm 5400$ | $\pm 7100$ | 76 | 24 |
| 20 | $\pm 49$ | $\pm 135$ | $\pm 3300$ | $\pm 4000$ | 64 | 21 |

Table VII. Effect of vertical motion on tensile column forces and displacements for a RC frame. No tension occurs under only horizontal loading. Calculated using data from Koukleri[25]

| Column | Tensile force (kN) $(H+V)$ | Tensile displ. $(H+V)$ (mm) |
|---|---|---|
| 1st storey exterior | 475 | 1·90 |
| 4th storey exterior | 350 | 1·30 |
| 8th storey exterior | 150 | 0·60 |
| 1st storey interior | 500 | 1·95 |
| 4th storey interior | 750 | 3·75 |
| 8th storey interior | 210 | 0·70 |

Table VIII. Effect of vertical motion on compressive column forces for a RC frame. Calculated using data from Koukleri[25]

| Column | Compressive force (kN) $H$ | Compressive force (kN) $H + V$ | Contribution of vertical motion to total axial force (%) |
|---|---|---|---|
| 1st storey exterior | 1500 | 1750 | 14 |
| 4th storey exterior | 750 | 1250 | 40 |
| 8th storey exterior | 125 | 350 | 64 |
| 1st storey interior | 1450 | 2500 | 42 |
| 4th storey interior | 800 | 2525 | 68 |
| 8th storey interior | 215 | 1200 | 82 |

Table VIII shows that the contribution of vertical motion to axial column forces in RC frames is again significant, as discussed earlier for steel buildings. In the light of the above observations, the possibility of direct failure due to axial loads, either in tension or compression, arises. Steel frames are particularly vulnerable in this respect due to local or global instability. Moreover, building frames designed with a 'capacity design' approach will be more prone to this type of failure at mid-height of internal and upper storey columns, since critically detailed regions are provided only at member ends, an observation consistent with field evidence mentioned earlier.

Moreover, it is known that high compressive or tensile loads reduce the capacity of concrete columns in shear and flexure.[26] The above fact suggests that not only the demand imposed is increased, but also that the capacity of the system may also be reduced due to the inclusion of the vertical component. In addition, the increase of compressive forces reduces the rotational ductility of columns,[26] while significant reduction in the axial force leads to loss of passive column confinement.[27]

*3.1.3. Effect on response modification factors (q or R).* Owing to the compendium of evidence collected at Imperial College and elsewhere, preliminary studies of the effect of vertical motion on behaviour factors were undertaken. Georgantzis[23] performed non-linear analysis of a coupled shear wall 8-storey 3-bay RC frame designed according to EC-2/EC-8 for an acceleration of 0·15 g.[22] The strong-motion records for this analysis were selected on the basis of $a/v$ ratio and predominant period coincident with that of the fundamental horizontal period of the structure. It was indicated that the inclusion of vertical motion, even when this is constrained to the $V/H$ ratio implied by modern codified values, may lead to significant reductions of the behaviour factor of up to 30 per cent. This is based on the definition of the yield acceleration causing an overall displacement corresponding to a secant stiffness at 75 per cent of the ultimate push-over load, as commonly used in the literature. The ultimate limit state is defined by column shear failure (according to the model of Priestley *et al.*[28]) or a 3 per cent drift limit, whichever precedes. This is discussed further below. The results are summarized in Table IX.

It is observed that the inclusion of vertical motion results in upper-storey failure, a result consistent with the higher variation of axial forces in upper storeys and field observations. Moreover, it is clear that column shear failure becomes the controlling factor for ultimate response when the vertical component is included in the analysis. Such critical behaviour is illustrated in Figure 23 for the response to the 1971 San Fernando record at Castaic Old Ridge.

In the above, the shear failure criterion proposed by Priestley *et al.*[28] was implemented in a time-step fashion. Supply and demand indicate clearly that in the presence of vertical earthquake input, shear failure is likely. The exceedance of the demand over the supply is repetitive and is not of a very high frequency to render it ineffective. The fluctuation of the shear capacity is of a period comparable to the vertical vibration period of the structure (0·1 s; Table IV).

Table IX. Reduction of behaviour factor and associated failure modes for an 8-storey coupled wall RC frame designed according to EC-2/EC-8. Reproduced using data from Georgantzis[23]

| Record | $V/H$ | $q$-factor reduction (%) | Ultimate limit state |
|---|---|---|---|
| Imperial Valley: Bonds CNR | 0·62 | 8·5 | Shear failure: external 7th storey column of coupled frame |
| San Fernando: Castaic Old Ridge | 0·63 | 29·3 | Shear failure: external 7th storey column of coupled frame |
| Kobe: Fukushima | 0·88 | 50·5 | Shear failure: internal 3rd storey column of MRF |
| Northridge: Arleta | 1·79 | 41·3 | Shear failure: external 7th storey column of dual frame/MRF |

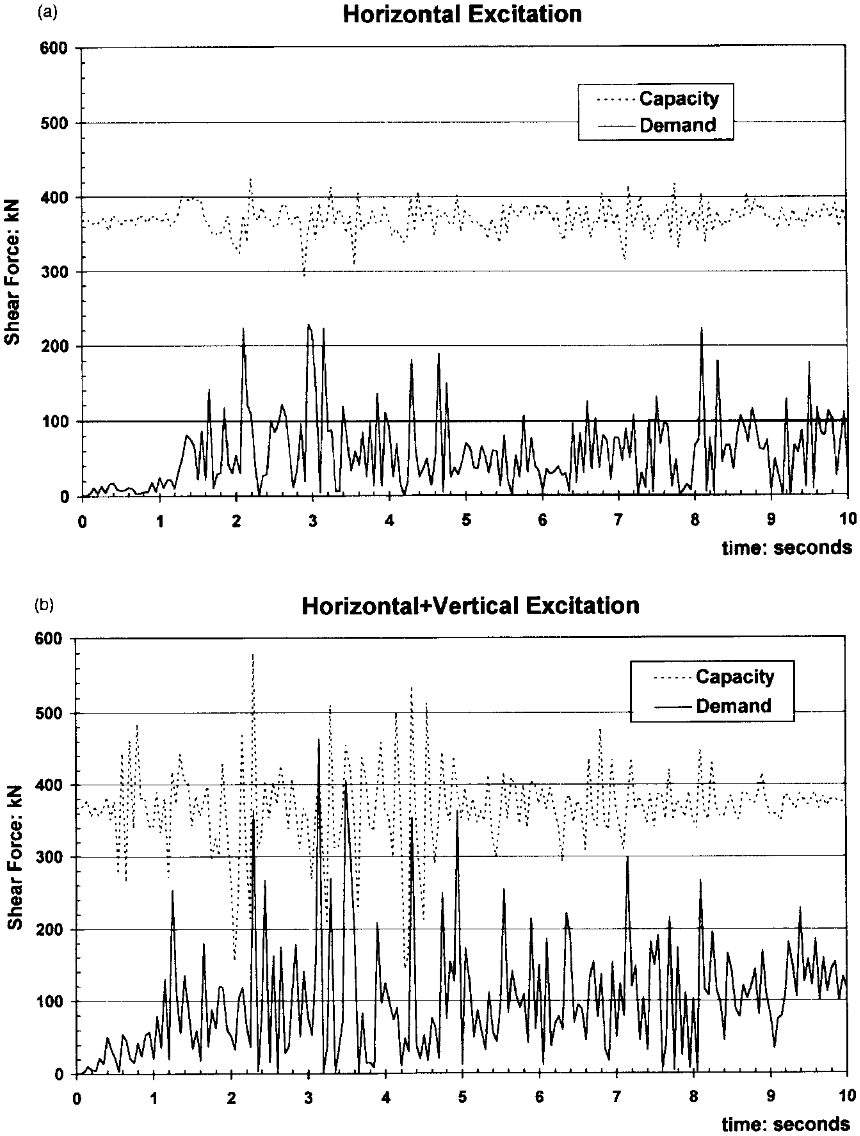



Figure 23. Effect of vertical earthquake motion on shear response of RC columns. Response to the 1971 San Fernando record at Castaic Old Ridge. Shear capacity/demand time-history of critical 7th storey column of dual frame at attainment of 3 per cent inter-storey drift in structure. Shear capacity according to the model of Reference 28. Reproduced using data from Georgantzis[23]

*3.1.4. Effect on response of steel frames.* Following the reports of damaged steel buildings in Northridge analysis of a typical 6-storey 3-bay moment resisting steel frame was undertaken by Broderick *et al.*[10] Typical wide bay spans of 8 m, 12 m and 16 m are investigated in the analysis. The frame is designed for an acceleration of 0·25 g. The particular structure is subjected to various combinations of horizontal and vertical components of the Santa Monica and Arleta records.

The results of this analysis suggest that vertical motion does not significantly influence transverse response parameters, i.e. inter-storey drift. However, an increase in column rotation ductility demand of the order of 12 per cent is observed and attributed to the occurrence of a lower yield rotation. More affected is the response of beams in terms of vertical excitation. It is observed that for long span structures vertical beam vibration modes excited by vertical motion may experience significance response amplification, even when the fundamental vertical mode of the frame as a whole does not coincide with that of the input motion. the amplitude of the obtained vibrations is such as to imply the imposition of a large number of cycles close to and exceeding yield at the connection. By allowing for a natural decay of the vibrations after the end of the strong-motion, more than 100 cycles are counted in certain cases, enough for low-cycle fatigue to become significant and cyclic deterioration to occur in typical connection elements. Such response only takes place if the vertical component is included in the analysis.

In figure 24, the rotation versus time plots for beam lengths of 8m, 12m and 16m are shown. It is clearly indicated that failure of the connection for the 16m beam is highly likely, since rotation capacity is typically in the range of 2·5–3·5 per cent. Such response, in combination with poor detailing at areas of high stress concentration at beam column connections, may be a serious contributory factor to connection damage observed in several steel buildings in Northridge and Kobe.

*3.2. Case studies of bridges*

*3.2.1. La Cienega-Venice Undercrossing Collector Distributor 36.* An analytical study of the bridge structure damaged in the Northridge earthquake has been undertaken to identify the factors that lead to its partial collapse. A 3-D non-linear analysis was carried out in order to evaluate critical response parameters.[29] Moreover, although not explicitly included in the analysis, a biaxial shear model including the effect of axial force interaction[28] was used to monitor the shear response of the piers. In the analysis involving the three components of ground motion the closest available record, the one obtained in the Santa Monica City Hall, was used. It is noted that this record was obtained some 10 km west of the structure under examination

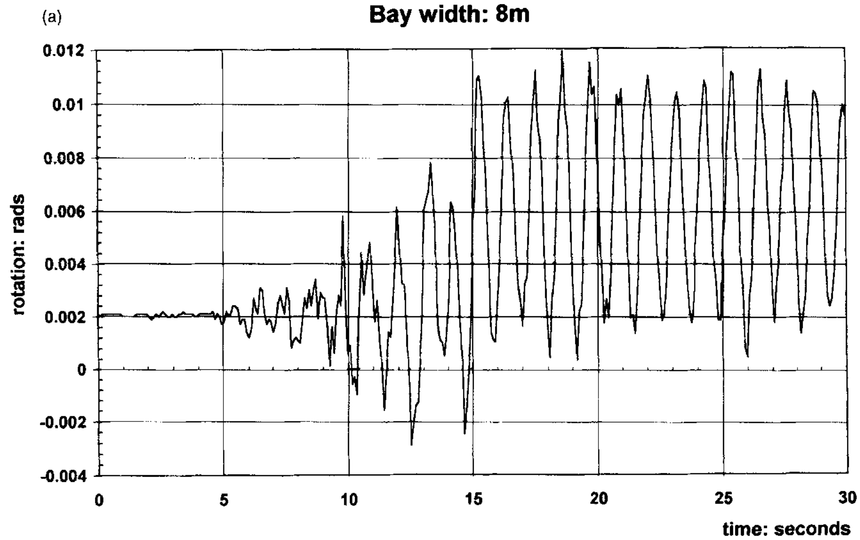
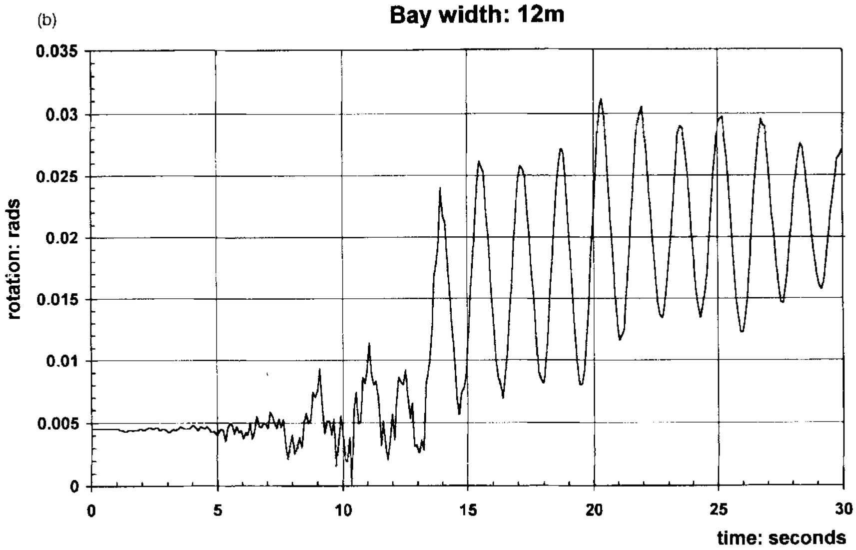



Figure 24. Second storey beam rotation time-histories for different bay widths of a steel 6-storey, 3-bay frame, subjected to the Northridge Santa Monica record. Reproduced from Broderick *et al.*[10]



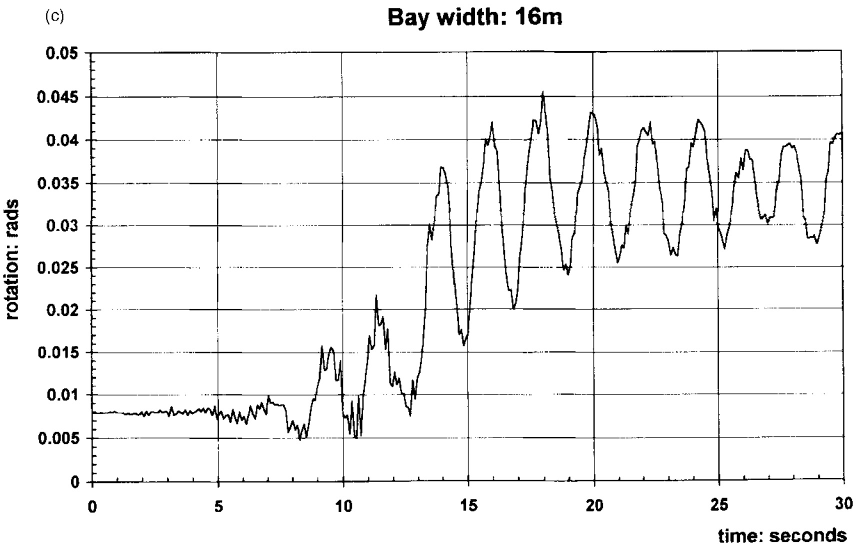




Figure 24. (continued)

and possesses horizontal acceleration peaks of 0·90 g and 0·41 g, while the vertical component peak is 0·26 g. It is stressed that the maximum accelerations experienced by the structure were of the order of 0·3 g for horizontal and 0·22 g for vertical motion, as indicated by recordings obtained from nearby stations.[11] Therefore, the used record is not necessarily the most critical from a vertical-to-horizontal peak motion viewpoint.

The analysis showed that moderate rotation ductility demands, within the available limits of supply, were imposed on pier 6 despite the spectacular failure experienced.[29] On the contrary, the shear response of piers 6 and 8 was more demanding and shear failure by a margin of 15 per cent was predicted when the average axial force over the time-history was used to assess the shear capacity.[29] However, the minimum axial force experienced by piers 6 and 8 was 58 per cent and 70 per cent of those assumed in the assessment of shear capacity.[29] Moreover, the time-history plots for biaxial shear response of the piers exhibit a peak at the same time when the axial force was at a minimum, approximately 9·75 s into the time-history.[29] The axial force

time-history and the effect of different components of input motion on axial force response are shown in Figures 25 and 26, respectively.

Figure 25 shows that the compressive axial force imposed on pier 6 at the time of peak shear demand is at a minimum. It is also significantly smaller than the axial force to which the other two peirs are subjected. Hence, the reduction of shear capacity for pier 6 is larger than the other two piers. This is in perfect correlation with the observed failure mode, since pier 6 suffered the most severe damage. In addition, Figure 26 indicates that the resulting fluctuation of axial load for pier 6 is primarily due to vertical motion. Hence, the damage inflicted on this particular structure is strongly attributed to vertical motion and in particular to its role in undermining the shear capacity of the damaged piers.

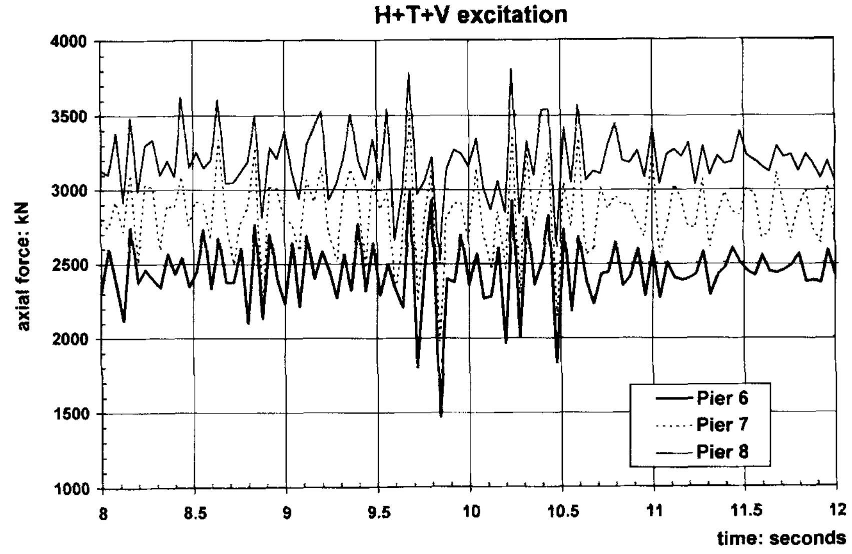



Figure 25. Pier axial compression responses to all three components of Santa Monica record. Reproduced from Broderick *et al.*[29]

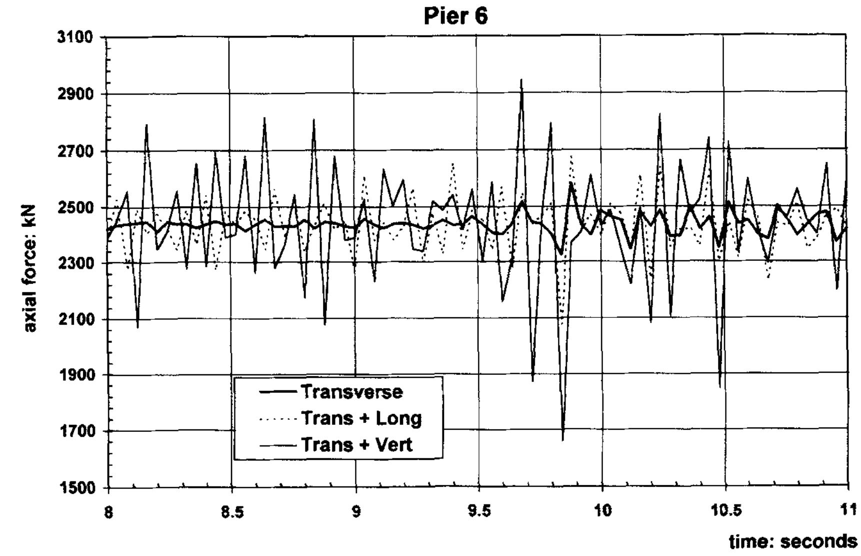



Figure 26. Effect of different components of motion on axial force response. Reproduced from Broderick *et al.*[29]

*3.2.2. Curved asymmetric bridge.* A study of a hypothetical multi-span curved asymmetric RC bridge was undertaken at Imperial College in collaboration with the University of Pavia.[30] The bridge was designed to withstand a 0·5 g event. The input motion, to which the response shown in Figure 27 was obtained, is composed of an artificially generated record consistent with the EC-8 soil-class B spectrum. The same record is applied in all three directions, with the input motion for vertical excitation scaled by a factor of 2/3.

Preliminary results from a 3-D nonlinear time-history analysis, which includes detailed modelling of abutments and soil-foundation-superstructure interaction show that under the design event of 0·5 g shear failure occurs in the squat pier only if the vertical component is included in the analysis. Again, the shear failure criterion proposed by Priestley *et al.*[28] is used. It is evident from Figure 27 that for the particular

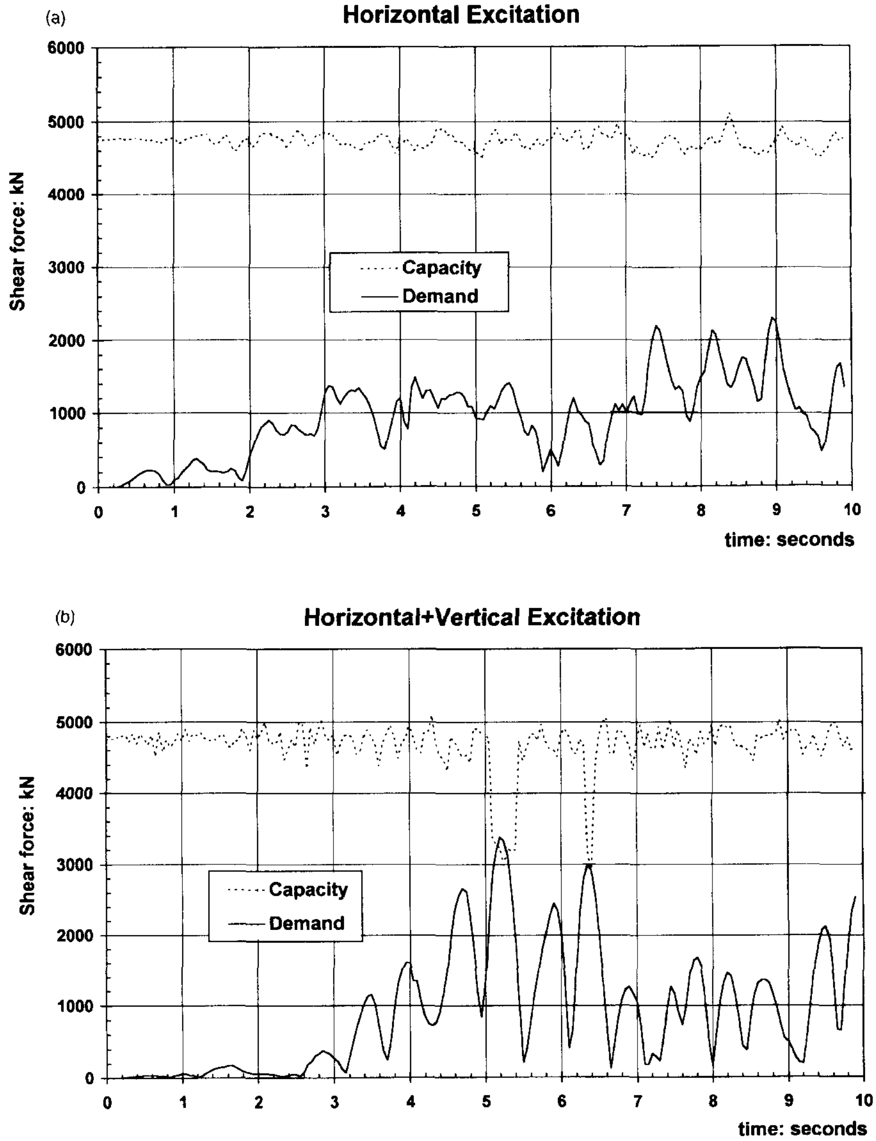



Figure 27. Shear capacity and demand time-histories for curved RC bridge under horizontal and horizontal–vertical excitation. Produced using data from Dodd *et al.*[30]

structure the timing for minimum axial force and maximum shear demand coincide, hence resulting in shear failure. The force balance used to suggest that shear failure is likely is reasonable in view of the fact that it is not of very high frequency and corresponds to displacements significant to cause shear distress. Moreover, whereas the writers accept that displacement-based design is the way forward, existing seismic codes are entirely force-based; hence, the above assessment is consistent with existing code philosophy. Finally, the analysis was repeated using six sets of natural earthquakes (each comprising two horizontal and their compatible vertical component[5]). The results gave further confirmation that neglecting the vertical component in seismic assessment may lead to potentially unconservative observations.

*3.2.3. Hanshin Expressway.* An analysis of the structure which was discussed in Section 2.2.3 and shown in Figure 18 was undertaken at Imperial College.[16] The failure pattern cannot be attributed with certainty to neither shear nor flexural mechanisms, since the concrete cover spalled symmetrically and longitudinal reinforcement buckled over a small length at mid-height of several piers. In addition, the core concrete was nearly crushed, but did not disintegrate fully. It was this pattern of failure that motivated the Imperial College group to study this particular bridge.

The non-linear model of the bridge structure was limited to three piers (P-663, P-664, P665) and two spans, the pattern of which is repetitive and constitutes the whole structure. Soil-structure interaction modelling did not include piles, but solid springs with compliances representing an average soil stiffness were used at the bottom of the footing. To represent the restraining effect of the fill material, transverse soil springs were also inserted above the footing up to the height of the fill. The first 30 s of the Kobe JMA record were employed in the analysis. The three components were used, but separate analyses were undertaken with and without the vertical component to ascertain the effect of vertical earthquake motion on this structure.

In the dynamic analysis response values were recorded at the location of reinforcement cut-off, i.e. at the location of the observed damage. Again, the shear failure criterion proposed by Priestley *et al.*[28] was implemented in a time-step fashion in order to assess the shear response of the three piers. Figure 28 shows the shear response of the piers to horizontal as well as to horizontal and vertical excitation. Figure 29 shows the axial force of the piers under the same excitations.

Figure 28 indicates that firstly the effect of vertical motion on shear response is negligible. This is due to the negligible effect of the vertical force contribution compared to the concrete and reinforcement contributions for the very large section dimensions. The plots indicate that the demand exceeds the supply for piers P-663 and P-664, albeit for a small margin and a short duration. This correlates perfectly with the observed damage pattern. However, the failure mode observed is not shear.

In order to investigate further the cause of the damage, the flexural response is examined. For all piers the maximum moment is less than capacity throughout the time-history.[16] In addition, pier P-665 is more stressed in bending than the other two. Moreover, the ductility demand imposed on all three piers is rather low.[16] The above are not in agreement with the observed damage pattern and hence do not provide a robust explanation for the mode of failure.

Finally, the axial force response is investigated. Figure 29 shows the axial force time-histories for the three piers under horizontal and horizontal and vertical excitation. It is clear that the fluctuation of axial forces under horizontal excitation is small. On the contrary, when vertical motion is included in the analysis this fluctuation becomes much larger. The magnitude of the oscillations is of the order of $\pm 70$ per cent of the static axial load for piers P-663 and P-664, but do not exceed 10 per cent of the compressive axial force capacity. The results for pier P-665 show clearly that it is less affected than the other two piers, since the magnitude of the axial force oscillations is much less and their nature is far less repetitive. This correlates perfectly with the observed failure mode since the most damaged piers are the ones with the highest and most repetitive fluctuation of the axial force. In this respect, the fluctuation of axial load on the piers could have been sufficient to cause cover spalling and buckling of the longitudinal reinforcement, hence further reducing bending and shear capacities. Minor site effects may also have played a significant role, in terms of long-term relative settlement which will tend to alleviate the axial load off the settled pier and increase it on adjacent piers.

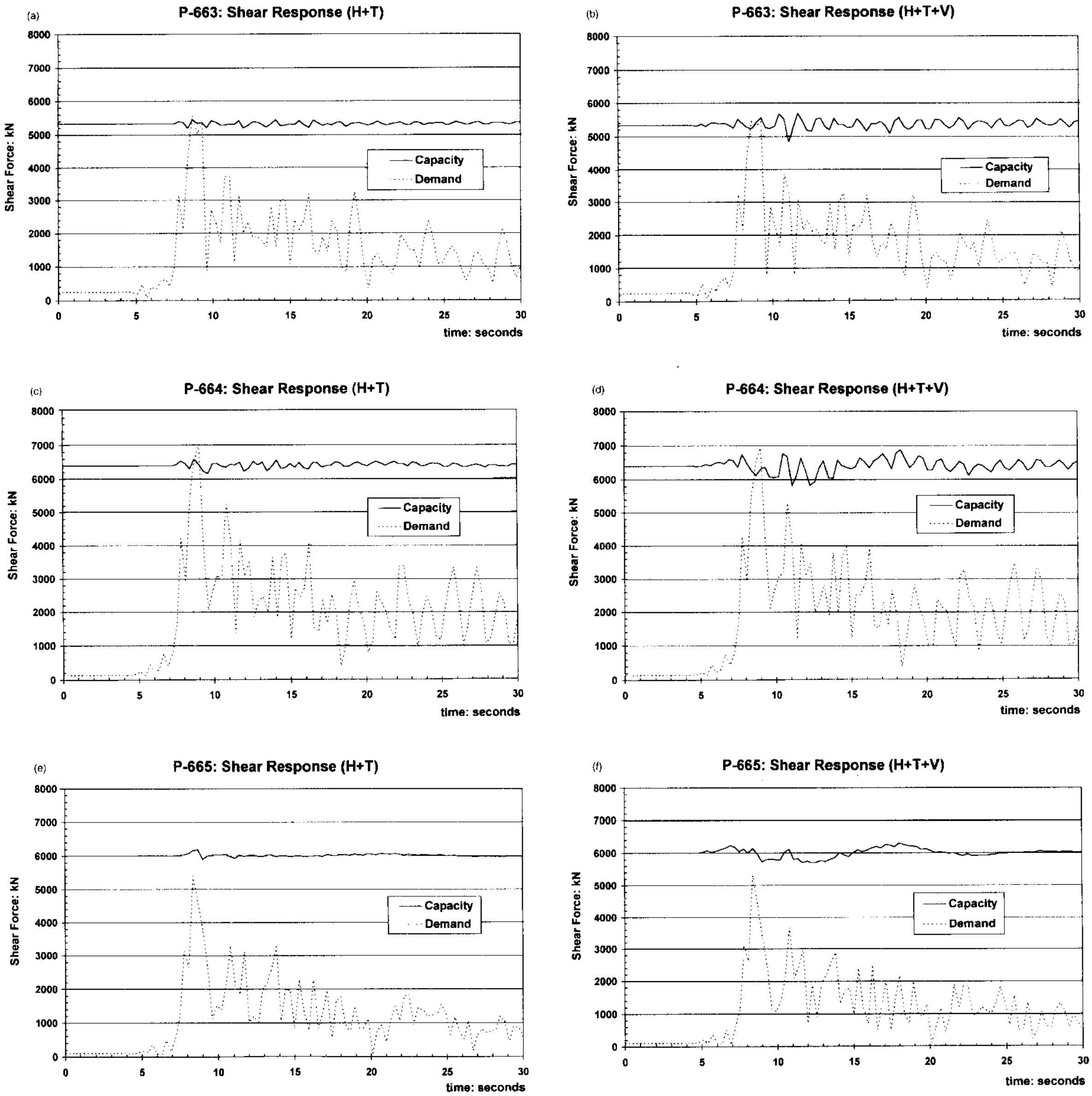



Figure 28. Shear capacity and demand versus time at reinforcement cut-off section for piers P-663, P-664 and P-665 of Hanshin Expressway. Demand calculated as vectorial sum of the two transverse components. Graphs on the left show the response to the two horizontal components, while graphs on the right show the response to all three components of the Kobe JMA record. Data from Elnashai *et al.*[16]

## 4. CONCLUSIONS

A compendium of field observations and analytical results indicate that certain failure modes are more convincingly attributable to high vertical earthquake-induced forces. In addition to the possibility of compressive overstressing or failure due to direct tension, vertical motion may induce failure in shear and flexure. Under reduced compression or mild tension, the contribution of concrete to shear resistance is eroded, thus many observed shear failure modes may have underlying vertical motion effects. In addition, the moment capacity and ductility of RC columns is reduced. Hence, there may be cases where apparently flexural failure modes are precipitated by axial force fluctuations.

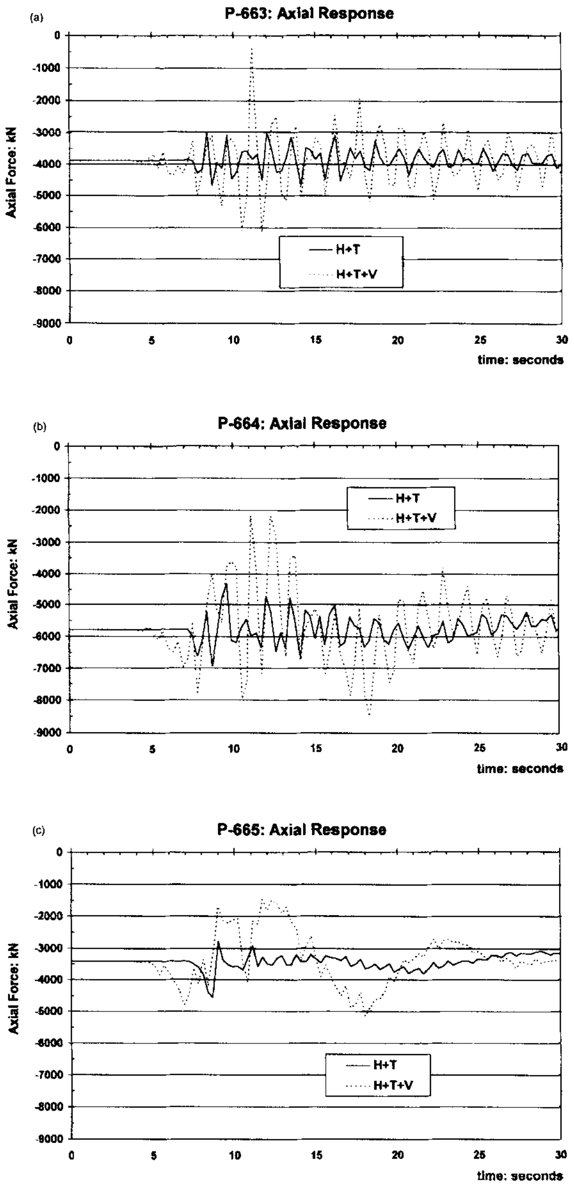



Figure 29. Axial force versus time at reinforcement cut-off section for piers P-663, P-664 and P-665 of Hanshin Expressway. Response to different component combinations of the Kobe JMA record. Data from Reference 16

The observational and analytical evidence presented above collectively point towards the importance of including vertical motion in earthquake resistant design and analysis. Simple procedures for the inclusion of the vertical component in the design process are therefore urgently needed.

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